553

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PROCEEDINGS

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DISCUSSION
OF PROCEEDINGS - SEPARATES

315, 325, 328, 385, 416, 417, 435

SOIL MECHANICS AND FOUNDATIONS
DIVISION

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DISCUSSION OF THE STRUCTURE OF INORGANIC SOIL
PROCEEDINGS—SEPARATE NO. 315

T. WILLIAM LAMBE,[1] A.M. ASCE.—The writer has studied Rosenqvist's excellent work and has had extended personal discussions with him in his Norwegian laboratory. As Rosenqvist states, there is substantial agreement between the writer and him on the nature of soil structure. An unimportant difference is the writer's belief that the thickness of immobilized water around a soil colloid is uniquely related to its diffuse double layer. The writer also considers the zeta potential less fundamental than does Rosenqvist. Truly inter-particle repulsion is determined by some potential between the particles but the effective potential has not been shown, to the writer's satisfaction, to be the zeta potential. The location of the slippage surface at which the potential is equal to the zeta potential is controversial.

The writer agrees that experimental data to support his belief, that the diffuse double layer and immobilized water layer are uniquely related, are needed. He can find supporters of Rosenqvist's view nearer home than beautiful Norway, namely, in his own M.I.T. laboratory.

Rosenqvist has presented experimental evidence to support much of the writer's structure theory, especially the part on particle orientation. It is hoped that Rosenqvist can and will employ his microscopic technique to obtain photographs of the same clay in both undisturbed and remolded states. The importance of particle orientation to the engineering properties of fine grained soils becomes increasingly evident.

Skempton emphasizes that important changes in the properties of a remolded soil can be caused by leaching. Skempton's, Rosenqvist's, and the writer's data all suggest that the reduction in the strength of an undisturbed clay caused by leaching would normally be less than 30%. The effect on other properties of undisturbed soil, especially permeability, may be considerably greater.

Skempton's Fig. 2 is most significant since it indicates in situ changes in soil properties. A soil cannot necessarily be considered "dead" or "inert" since it can be very much "alive" and sensitive to its environment.

The experimental observations of Barber are consistent with the writer's theory. The writer takes sharp issue with some of the concepts of structure proposed by Barber. Barber, for example, states (p. 401-9) "Swelling of clay masses is caused by straightening of bent particles. . . . ." Considerable data (e.g. Cornell reference in writer's paper) have been presented to show that the crystal lattice of a swelling soil mineral expands in the c direction with water pick-up. In fact, a close correlation between lattice dimension and thickness of water molecules has been presented. To accept Barber's view would require that many of the fundamental concepts of clay mineralogy be discarded. These concepts are too well substantiated to be ignored without convincing evidence to the contrary—Barber has not presented such.

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The writer, while unable to accept Barber's picture of thixotropy, agrees that the phenomenon can be explained by particle to particle contact. The writer has studied the original paper by Fahn et al, referred to be Barber, as well as the Proceedings of the British Mineralogical Society before which the paper was presented. Discussion at this meeting emphasized that there is controversy on the causes of thixotropy. Two points should be emphasized: (1) the short discussion on thixotropy was included by the writer to show that there are two entirely different uses of this term; (2) the explanation presented is widely, but not uniformly, accepted.

The writer appreciates the discussions to his paper: Rosenqvist presented some experimental evidence to substantiate part of the theory; Skempton emphasized the practical importance of soil structure; and Barber showed that the writer's theory has some opposition, a healthy situation. The writer believes, with increasing fervor, that soil structure is vital to an understanding of the behavior of fine grained soils, and hopes, therefore, that research and discussion on it will increase.

DISCUSSION OF BUILDING FOUNDATIONS IN SAN FRANCISCO
PROCEEDINGS—SEPARATE NO. 325

CHARLES H. LEE,[1] M. ASCE.—The writers, in contributing additional information regarding experience at certain buildings in San Francisco have opened up two subjects of great interest to foundation engineers. The first is the use of dynamic formulae and, in particular, the Engineering News-Record formula for determination of bearing capacity of piles. They conclude that the use of such formulae "would be not only erroneous, but actually dangerous." Recent experience of the author in connection with driving steel H-piles at the site of the Equitable Life Building, Sutter and Montgomery Streets, San Francisco, confirms this conclusion.

The generalized soil profile of the site is indicated on the accompanying table. The "rusty-brown silty sand lenses" penetrated between elevations -62 to -93 ft. are produced by ferriginous cementation and are very irregular in shape and unpredictable in location. They have been called "hardpan" at the Equitable Life Building site but correspond to the so-called "sandy 'kidneys'" encountered in pile driving operations at the PG&E Building referred to by the writers. The blue marine clay between elevations -93 to -116 ft. is very compressible and is the cause of appreciable settlement at buildings in the downtown district of San Francisco whose weight is sufficiently great to increase pressures at this depth. In order to obtain firm support, as well as avoid local overloading of the blue marine clay, the Equitable building is being supported on steel H-piles driven into the underlying rock. Above the hard rock is a zone of weathering in which the rock has softened, with progressive decomposition and internal shearing. In the upper portion of this zone the rock is a soft shale, merging at lower levels into broken, harder rock and, finally, into the sound hard rock condition. The softened Franciscan sandstone has test values in internal friction of 43°, cohesion of 700 p.s.f., and unconfined compression of 22 to 37 tons p.s.f.

The foundation plans indicated use of 14 x 14-1/2 inch, 102-lb. H steel bearing piles with design load of 135 tons. The specifications required that piles be driven into bedrock to a final resistance of 15 blows per inch with a single-acting steam hammer developing 50,000 ft. lbs. of energy per blow, but permitted modification by the Architect/Engineer after the first few piles had been driven. Application of the Engineering News formula for 15 blows per inch penetration resistance, using a 17,000-lb. hammer and 39-inch height of fall of hammer, gives a bearing capacity of

$$\frac{2 \times 17{,}000 \times 3.25}{(0.066 + 0.1)\ 2000} = 332 \text{ tons.}$$

The rational static method, following generally the concepts presented by John W. Dunham, M. ASCE, in his recent paper,[2] indicates a safe bearing capacity of 135 tons but only with penetration of piles into the weathered rock of at least 3.5 ft. Driving experience indicated that this degree of penetration could not be obtained until a penetration resistance of 30 blows per inch was attained, the average penetration at 15 blows per inch being less than 2.4 ft. At the rate of 30 blows per inch the Engineering News formula indicated a bearing capacity of 415 tons. Conditions for use of the static method of computation are favorable, including availability of data on physical properties of the weathered rock and minimum change in physical properties of the rock during driving due to small cross sectional area of the steel H-pile. The ra-

1. Cons. Eng., San Francisco, Calif.
2. Proceedings, A.S.C.E., Separate No. 385, 1954.

tional static method was, therefore, followed and a penetration rate of at least 30 blows per inch was required for all piles driven.

The use of Engineering News-Record formula, in this instance, would have resulted in inadequate penetration of the weathered rock, resulting in excessive unequal settlement of columns.

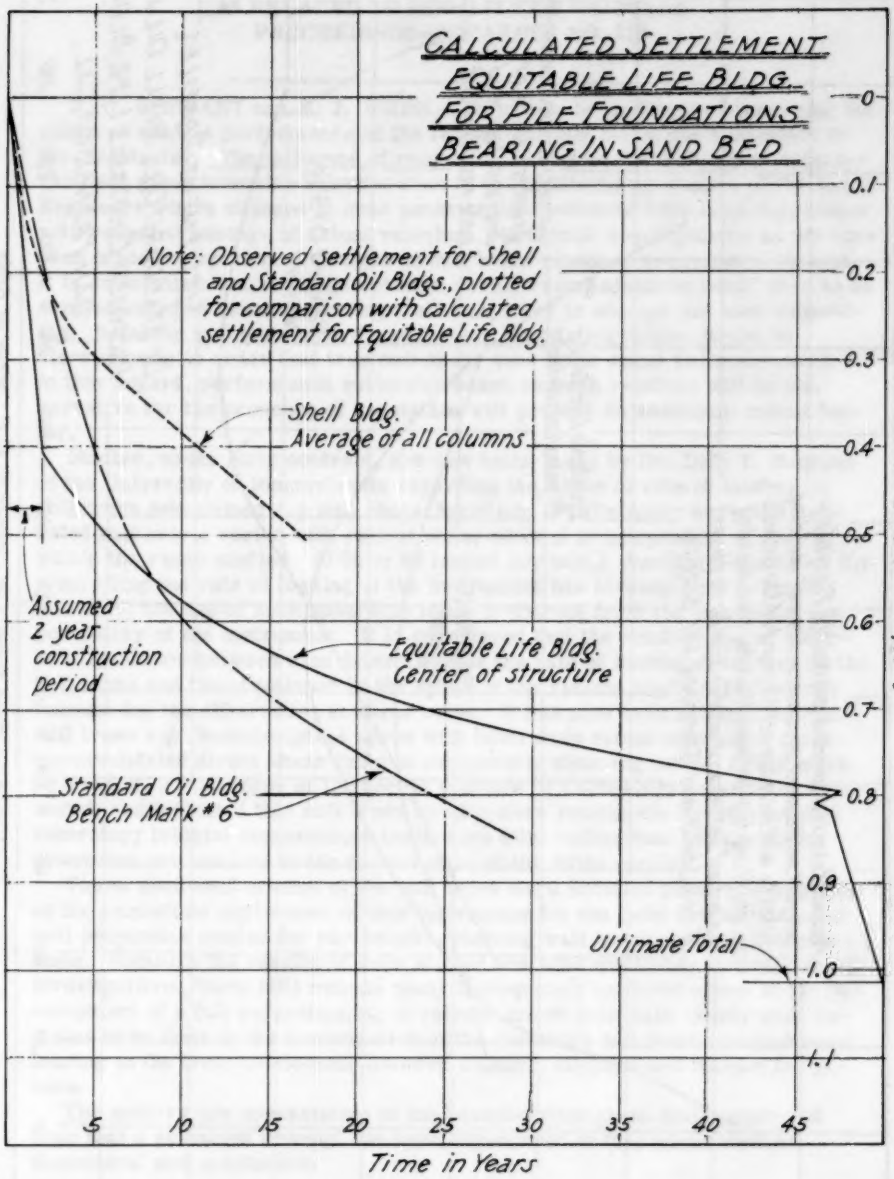
The other point of interest in the writers' discussion was their experience in estimating settlements for Standard Oil Building in San Francisco. The author has recently had opportunity to estimate settlement for the new 25-story Equitable Life Building, which immediately adjoins the Standard Oil Building and Annex. As a basis for computations there were available 11 complete consolidation characteristic curves, each with a range of load increments from 1 ton to 16 tons per sq. ft. and applying to materials in each of the five zones, B, C, D, E, and F. These characteristic curves were similar to those shown on Diagram 5 and were developed from laboratory tests made upon undisturbed samples from test borings taken with 3-inch thin-walled Shelby tube sampler. Computations were made for three soil profiles at the building site, prepared from 5 test boring logs, with additive soil pressure isobars superimposed for the building load assumed acting at elevation -44 ft. The bottom level of the mat foundation beneath the Standard Oil Building is at elevation -38 ft. The results are shown upon the accompanying diagrams of settlement profiles and settlement-time relationships, as computed for the Equitable Building and as historically observed for Standard Oil and Shell Oil Buildings. It is apparent from these diagrams that the computed and historic values are in reasonably close agreement. As stated by the writers, it is important in making such calculations that there be sufficient test borings and undisturbed test samples to provide a comprehensive view of subsurface conditions.

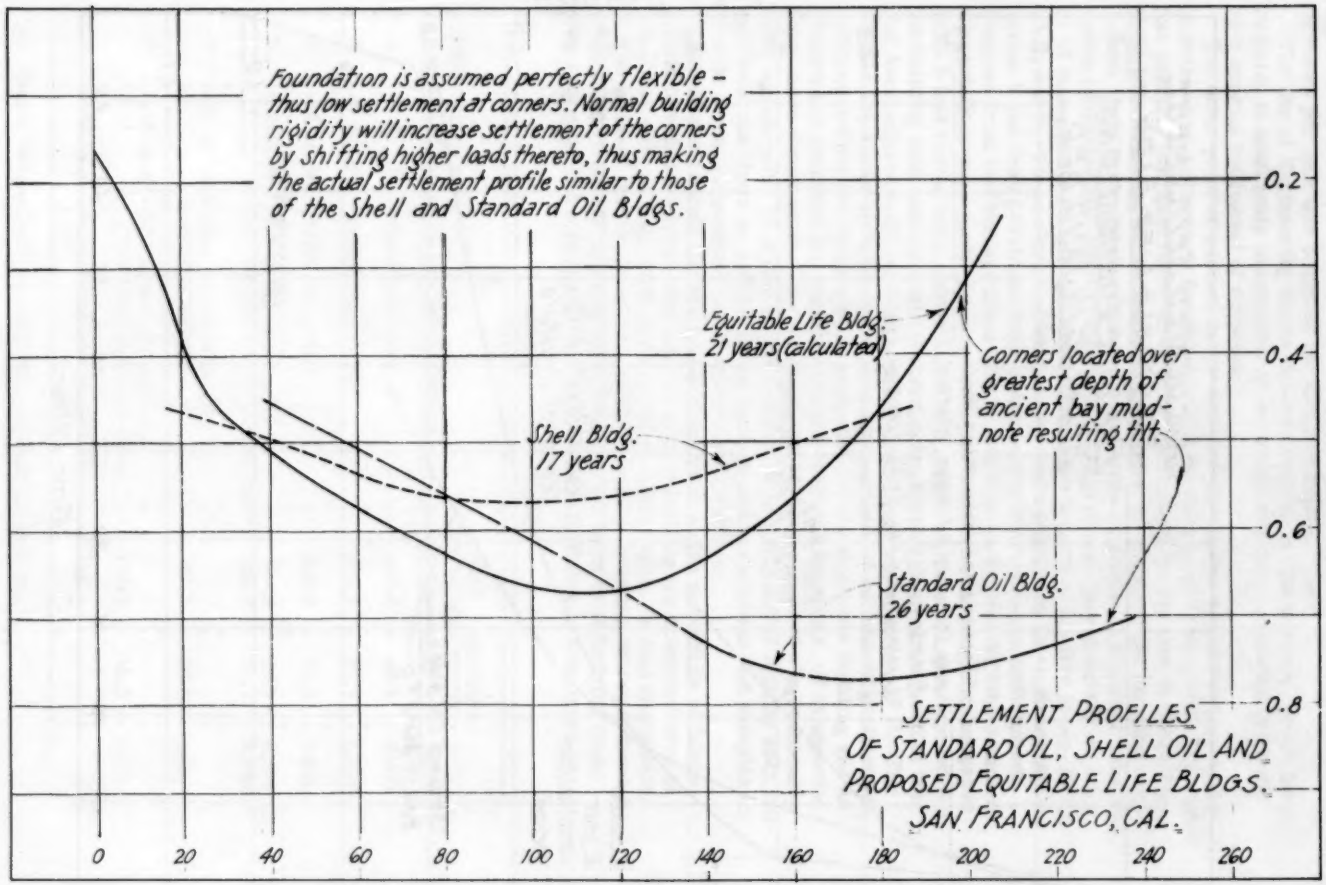
The author regrets the absence of discussion of the possibility of future plastic flow in the deeper blue marine clay underlying the downtown area of San Francisco, as suggested on Page 325-2 of his paper. In this connection, it is of interest that the foundation of the Equitable Building, which is located in the most congested building area, was carried down to underlying rock. This may establish a precedent for future large buildings in this area.

GENERALIZED SOIL PROFILE

ZONE	ELEVATION FT.	DESCRIPTION	UNIT DRY WEIGHT P.C.F.	MOISTURE CONTENT % DRY WT.	VOID RATIO
A	+9 to -8	Wind-blown sand	102.8	18.2	0.623
B	-8 to -18	Yellow sandy clay	113.0	17.5	0.470
C	-18 to -62	Brown silty sand	106.0	18.3	0.584
D	-62 to -93	Yellow sandy clay with	99.6	23.9	0.679
		rusty-brown silty sand lenses	103.7	24.5	0.610
E	-93 to -116	Blue marine clay with organic matter	72.1	48.4	1.34
F	-116 to -133 (a)	Decomposed and sheared Franciscan s.s. and shale	118.3	14.3	0.39
G	-133 to -	Hard Franciscan sandstone	155 ±		

(a) Varies between -107 to -115 ft. and -130 to -152 ft.


CALCULATED SETTLEMENT
EQUITABLE LIFE BLDG.
FOR PILE FOUNDATIONS
BEARING IN SAND BED
Note: Observed settlement for Shell and Standard Oil Bldgs., plotted for comparison with calculated settlement for Equitable Life Bldg.
Shell Bldg.
Average of all columns
Assumed 2 year construction period
Equitable Life Bldg.
Center of structure
Standard Oil Bldg.
Bench Mark # 6
Ultimate Total
0
0.1
0.2
0.3
0.4
0.5
0.6
0.7
0.8
0.9
1.0
1.1
Settlement in Feet
5
10
15
20
25
30
35
40
45
Time in Years


Foundation is assumed perfectly flexible – thus low settlement at corners. Normal building rigidity will increase settlement of the corners by shifting higher loads thereto, thus making the actual settlement profile similar to those of the Shell and Standard Oil Bldgs.
Equitable Life Bldg. 21 years (calculated)
Corners located over greatest depth of ancient bay mud- note resulting tilt.
Shell Bldg. 17 years
Standard Oil Bldg. 26 years
0.2
0.4
0.6
0.8
Settlement in Feet.
0
20
40
60
80
100
120
140
160
180
200
220
240
260
Distance in Feet from S.W. Corner to N.E. Corner of Bldg.

SETTLEMENT PROFILES OF STANDARD OIL, SHELL OIL AND PROPOSED EQUITABLE LIFE BLDGS. SAN FRANCISCO, CAL.

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DISCUSSION OF TRAFFICABILITY OF SOIL AS RELATED TO MOBILITY OF VEHICLES PROCEEDINGS—SEPARATE NO. 328

R. C. STEWART and S. J. WEISS.—There can be no dispute concerning the effect on vehicle performance of the factors brought out in the discussion by Mr. McMaster. The influence of remolding of soil on the mobility of vehicles has been given intensive consideration in the trafficability studies of the Army Engineers where changes in cone penetration resistance have been correlated with repeated passage of actual vehicles. No formal investigations as yet have been made on concurrent changes in soil truss readings or moisture content. It is anticipated that readings obtained during reconnaissance would need to be supplemented at intervals during traffic in order to account for such remolding. In taking soil tests with the soil truss, vegetative matter would be cleared away in order that true soil shear take place under the instrument. In this regard, performance estimates based on such readings will be conservative for the presence of vegetation will provide an additional safety factor.

Studies, under Navy contract, are now being made by Dr. Dale T. Harroun of the University of Pennsylvania regarding the effect of rate of loading on soil truss determined c,ϕ soil characteristics. Preliminary work has indicated that only c varies with rate of shear while ϕ is independent of speed within the range studied. (0.05 to 60 inches per min.) Auxiliary apparatus for controlling the rate of loading of the instrument has already been devised. However, the use of such apparatus tends to detract from the simplicity and portability of the instrument. It is considered that the establishing of a correlation factor between c as determined at the rate of loading developed by the soil truss and that developed at the speed of the vehicle would satisfactorily account for the differences in shear rates. It has also been determined that soil truss c,ϕ characteristics agree with laboratory strain controlled quick-unconsolidated direct shear tests at comparable shearing rates. Other work by Prof. F. J. Converse of California Institute of Technology, also Navy sponsored, has indicated that soil truss results show reasonable agreement with laboratory triaxial compression tests when total rather than intergranular pressures are utilized in the construction of the Mohr circles.

These additional studies of the soil truss were initiated primarily because of the immediate application of this instrument for the field determination of soil properties needed for earthwork, retaining wall and foundation computations. Although the results of such studies are also applicable to trafficability investigations, there still remain many inadequately explored areas in the development of a full understanding of vehicle action over soil. Much work remains to be done in the investigation of the elasticity and plasticity problems leading to the true relationship between sinkage, slippage and motion resistance.

The authors are appreciative of the consideration given their paper and hope that a sustained interest has been engendered in this application of theoretical soil mechanics.

KUANG-HAN CHU,[1] A.M. ASCE.—A semirational method for the determination of the length of friction piles has been presented by the author. The method is subjected to criticism on the following two points: (a) The pile length depends primarily on the selection of the depth of nonsupporting strata. Moreover, in striving for simplicity, the friction on a pile in the depth of h within the supporting strata is neglected in deriving Eq. (12). Thus, when $H_1 = H_2 = 0$, Eq. (12) gives $h = \infty$. The selection of the depth of nonsupporting strata which determines the pile length depends primarily on judgment. (b) It is assumed that the friction between the pile and the soil is greater than the shearing strength of the soil. This is hardly supported by test data. In fact, the author's assumption in using shearing strength of soil for cohesive soils and passive pressure multiplied by the coefficient of friction of $\tan\phi$ for noncohesive soils gives such high values of skin friction that these values seem to be hypothetical. For instance, in stiff clay, N = 40, the shearing strength is assumed to be 0.133N = 5.33 ksf; while Terzaghi & Peck[2] give ultimate skin friction as 2.0 ksf. Similarly, in noncohesive soils, the discrepancy between author's values and Terzaghi's are even greater.

In spite of the foregoing criticism, the author does contribute a semirational method which, if used carefully, may give reasonable results. The following are some suggestions on simplification of the author's formulas.

For cohesive soils; $q = \frac{8N}{30}$. $C = \frac{q}{2}$.

For noncohesive soils;

$K = \frac{2}{3}(0.75 + 0.1N)$ for round grains of uniform size.

$K = 0.75 + 0.1N$ for round grains well graded or sharp grains of uniform size.

$K = \frac{3}{2}(0.75 + 0.1N)$ for sharp grains well graded.

Eq. (11), $h = \frac{15(P-0.25Na)}{NO}$

Eq. (12), $h = \frac{32(P-0.20Na)}{(1.6H_1 + H_2)KO}$

Eq. (13), $h = \frac{\frac{P}{p} - a}{1.8r}$

B. A. KANTEY,[3] A.M. ASCE.—Like so many engineers, the author has come up against the problem of rational methods of design for piled foundations. Judging from his paper, he has rejected dynamic formulae, and has developed a method based on static formulae. He has, however, hit on the weakness of

1. Structural Designer, D. B. Steinman, Cons. Eng., New York, N. Y.
2. "Soil Mechanics in Engineering Practice," by Karl Terzaghi and R. B. Peck, John Wiley & Sons, Inc., New York, N. Y., 1948, p. 463.
3. Cons. Engr., Kantey & von Geusau, Cape Town, South Africa.

the static formulae methods in that, for the sake of economy, design has to be based on a comparative insufficiency of results when attempting to evaluate the load carrying capacity of the piles from the results of tests on undisturbed samples.

In order to overcome these deficiencies, the author has proposed a method of design in which the shear characteristics are obtained from the results of the "standard penetration test" allied, where necessary, to tests on undisturbed samples. That the author has evolved a successful method is evident from the results he quotes and, in the face of such evidence, one is hesitant of offering criticism. The writer considers, however, that there are certain inherent disadvantages in the proposed method and it is possible that, with more widespread use, greater discrepancies between design and actual pile lengths will appear.

These disadvantages lie in the fact that the "standard penetration test" is itself a dynamic test and, as such, should be no more accurate than dynamic formulae. In addition, the results of the test are transferred to an empirical formula relating penetration resistance to bearing capacity. This automatically implies some form of scatter about the mean results given and, while the errors may cancel out giving good results, these errors may also be cumulative, with a consequent large discrepancy between design and practice.

Some years back the same problem of rational pile design arose in South Africa and the writer was given the task of evaluating the various methods available. The findings were published in 1950[4] and, as a result of this task, the writer became convinced that the main future for rational piling design lay in the development of in-situ methods of determining the shear characteristics of the subsoil. Tools for such methods were developed in Eurpoe and have been described in the proceedings of all three International Conferences on Soil Mechanics and Foundation Engineering. These tools consist of the deep-sounding apparatus developed in Holland and Belgium for the more granular types of soils and the Swedish Vane Shear Apparatus for cohesive soils. Since the method of obtaining the results approximate most closely to static loading of the sub-soil through piled foundations, the writer considers that the degree of accuracy and reproducibility of results warrants their favourable consideration. The use of such methods is becoming more and more standard practice in Europe and South Africa and the writer would like to commend these methods to the author for his consideration.

JOHN A. FOCHT, JR.,[5] A.M., ASCE.—The author as a structural engineer is well-acquainted with materials having clearly defined and easily determined properties. But the strength and deformation characteristics of soils may not only vary from site to site but also may be altered by time and the construction procedure at any one site. As Prof. Tschebotarioff states in the preface to his book,[6] "the strong element of art in all soil and foundation engineering work should not be overlooked. The acknowledgement of its existence is necessary for the understanding of the present status of this field of knowledge and endeavor." It would appear to the practicing soils engineer that the design of pile foundations as presented has been over-simplified for the sake of having a few equations.

4. B. A. Kantey. "Significant Developments in Sub-surface Exploration for Piled Foundations." Trans. S. A. Inst. of C. E. Vol. I, No. 6 & Vol. 2, No. 12.
5. Associate Engr., Greer & McClelland, Cons. Foundation Engrs., Houston, Tex.
6. Tschebotarioff, Gregory P. "Soil Mechanics, Foundations, and Earth Structures." McGraw-Hill Book Company. 1952.

A structural engineer who is "dissatisfied with the tools he finds available in connection with pile foundations" has another alternative not listed in the Introduction. If engaged in private practice, he may seek assistance from competent soils engineers, particularly on large projects. The Army Corps of Engineers, Navy Bureau of Yards and Docks, and the Bureau of Reclamation have large staffs of personnel who should be able to aid any engineer in the Federal Government with foundation problems.

The following of local experience will prevent the occurrence of a grevious mistake. But past experience may lead to overly conservative and expensive designs. In many instances, piles have been used merely because the building down the street is supported on piles.

In the last paragraph of the Introduction, the author points out the limitation of dynamic pile driving formulas and "the need for more specific design and construction procedures." There is considerable literature published verifying the applicability of laboratory tests on undisturbed soil samples to the design of pile foundations.[7,8,9,10] The results of a single pile load test at a site may be interpolated by use of soil studies to other locations at the same site, or different size piles. Actual data on the shear strength, density, and deformation characteristics of the soils warrant much more confidence than penetration resistance.

Condition 3 for a successful pile foundation is misleading as stated. A single pile does not fail, actually plunge into the ground, because the soil around the point is overloaded. It fails because the sum of the skin friction and end-bearing capacity of the soil have been exceeded. Excessive settlement may result if the soil is stressed so as to cause consolidation. A group of piles may fail in a manner similar to a deep pier because the area of soil at the tips is stressed in excess of the bearing capacity.[11] The action of the group as a unit is not significantly different from the action of a single pile. It is proposed that condition 3 be rewritten:

3. Each pile group must be proportioned so that each individual pile will carry its load and that the total load on the group is safely transmitted to the soil by shear on the periphery and by bearing on the area of the group at the pile tips.

With this restatement, the design criteria without regard for settlement are then with respect to:

1. Structural capacity of each pile
2. Friction and end-bearing capacity of single piles
3. Friction and end-bearing capacity of pile groups
4. Bearing capacity of weaker soil strata beneath the pile tips

The seven simplifying assumptions listed and discussed very briefly deserve more consideration as they are the real basis of the design, rather than the equations derived using the assumptions.

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7. U. S. Waterways Exp. Sta. "Pile Loading Tests, Combined Morganza Floodway Control Structure." Tech. Memo. No. 3-308, January 1950.
8. Moore, W. W. "Experiences with Pre-determining Pile Lengths." Trans., ASCE, Vol. 114, p. 351, 1949.
9. Converse, F. J. "The Determination of Pile Bearing Capacity from Sub-soil Investigations and Laboratory Tests." Proc. 2nd Int. Conf. on Soil Mech., Vol. V, p. 142, 1948.
10. Masters, F. M. "Timber Friction Pile Foundations." Trans. ASCE, Vol. 108, p. 115, 1943.
11. Ghavem, Mohamed Fahmy. "Bearing Capacity of Friction Piles in Deep Soft Clay." Thesis for PhD, University of Illinois, 1953.

The first assumption that the pile receives no vertical support from unsatisfactory strata is valid. But the problem is in determining what is unsatisfactory. In example A, it is not apparent why stiff to very stiff clay (above 27-foot depth) would be unsatisfactory. It is also very likely that the loose sand in example B would become more dense during pile driving and could be considered as satisfactory supporting material.

The second assumption can be very overly conservative. The use of the lowest N value or shear strength within a given stratum is justified to make allowance for possible variations of soil properties in that stratum. But, complete elimination of slightly weaker higher strata, particularly if the foundation is all clay, in determination of pile bearing capacity will result in greater pile penetration than is required for safety. The stress-strain characteristics of each stratum should be considered before reaching the conclusion that "failure in the weakest stratum in the group will destroy the support of all higher strata and will throw more load on the other supporting strata than their properties can justify." It is often the case that soft clays develop their maximum strength at strains greater than the more stiff clays. The effect of consolidation of the softer clays must be considered, however, if the pile penetrates into stiffer material.

In place of the author's assumption 3, it is suggested that the unit pressure on the pile point be computed as the end-bearing capacity of the soil at that depth using equations given by Terzaghi and Peck.[12] This procedure for piles driven into sand has been verified by actual tests.[13] The effect of confinement on sand is to increase its shear strength and it is well-known that the bearing capacity of clay is considerably greater than its shear strength.

The fourth assumption as stated is taken to mean that the adhesion of the soil to the pile is greater than the shear strength of the soil a short distance from the pile. A pile driven into clay completely remolds the clay immediately around the pile. The final strength of this remolded clay may be greater or less than the undisturbed shear strength of the soil.[14] The very significant increase in skin friction with time[15,16] from a nominal value immediately after driving to the final strength must be considered on occasion.

The fifth assumption is common for piles in clay. However, Terzaghi and Peck[17] state: "In every sand both the average skin friction per unit area of contact and the point resistance increase with increasing depth." Further, the measurement of skin friction with depth in load tests[18] indicate that the shear stress does vary as the shear strength. Consideration of the increasing skin friction in sand will require less penetration. The field tests on piles in clay

12. Terzaghi, Karl, and Peck, Ralph F. "Soil Mechanics in Engineering Practice." John Wiley & Sons, 1948, p. 171.
13. Mansur, C. I., and Focht, John A., Jr. "Pile Loading Tests, Morganza Floodway Control Structure." ASCE Sep. No. 324, November 1953.
14. Taylor, Donald W. "Fundamentals of Soil Mechanics." John Wiley & Sons, 1948, p. 648-650.
15. Terzaghi, Karl, and Peck, Ralph B. "Soil Mechanics in Engineering Practice." John Wiley & Sons, 1948, p. 461.
16. Housel, W. S. Discussion of "Effect of Driving Piles into Soft Clay" by A. E. Cummings, G. O. Kerkhoff, and R. B. Peck. Trans. ASCE, Vol. 115, p. 339, 1950.
17. Ibid, p. 460.
18. American Railway Eng'g. Ass'n. Advance Report of Comm. 8. "Steel and Timber Pile Tests, West Atchafalaya Floodway, N.O., T. & M. Railway." Bul. 489, p. 149, September-October 1950.

indicate that the shear strength along the full length of the pile may be included in computation of the pile capacity.

It is believed that the sixth assumption should have the last clause deleted, so that superposition of the loads will be considered in determination of the vertical pressure beneath a pile group. The pressure beneath the center of a group will be greater than the pressure beneath the edge.

While the use of the "Standard Penetration Test" has had considerable acceptance, there are some doubts as to its reliability. Extended investigations by the Waterways Experiment Station, Vicksburg, Mississippi,[19] have indicated that the penetration resistance is not necessarily a function of the relative density of sand. The penetration resistance is materially affected by the depth of the boring, the location of the water table, the grain size of the material, and other factors. Careful observations have shown that there may be no correlation between the relative density and penetration resistance at all.[20] In the past, it has been considered that the relative density of a sand, and thereby its strength, is indicated by the driving resistance; it is on this basis that the design procedures outlined were developed. In view of the limitations which must be placed on results of the standard penetration test, the advocated design procedures are also limited.

The cost of securing a sufficient number of undisturbed soil samples in an adequate boring program supplemented by such laboratory tests as necessary will be only very slightly more than the cost of an equal number of penetration borings. Therefore, it would appear that a more rational, and nearly as economical (as related to investigation costs), design can be prepared utilizing undisturbed soil sampling and testing, rather than the standard penetration type boring.

JAMES F. MCNULTY,[21] A.M. ASCE.—The problem discussed by the author is one whose solution has eluded the engineer. Many methods, theoretical and empirical, have been presented for estimating the length of pile required to safely carry a given load. In spite of this, it appears that, at present, the best an engineer can expect to accomplish is to find a semi-empirical means of estimating the pile length which will check load test results for his particular local soil condition.

It is assumed that the author has succeeded in doing this although it would have been helpful had he included in his report some load test data for evaluation. Mr. Dunham, judging from remarks in the Field Procedure section, used the driving resistance of the pile as a check on his contract length; the unreliability of information derived from driving resistance data is too thoroughly documented to need further discussion. The writer wishes to emphasize that without load tests, the magnitude of the safety factor is unknown.

The correctness of Mr. Dunham's formula (1) depends mainly as to what degree the following two assumptions approximate true conditions:

a) uniform shear stress

b) shear stress at failure equals soil's cohesion value

Assumption (a) appears to be approximately true, based on a few scattered

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19. Waterways Exp. Sta., Vicksburg, Miss. "Standard Penetration Tests, Reid Bedford Sand, Mississippi River." Potamology Investigation Rep. No. 5-5, May 1950.
20. Mystkowski, A. "Sand Needn't Be Treacherous." Engineering-News Record, July 2, 1953.
21. Design and Construction Engr., National Advisory Committee for Aeronautics, Langley Field, Va.

load tests which utilized strain gages along the embedded length. No theoretical explanation is available except for a pure clay. Assumption (b) appears logical but load tests have failed to substantiate it in some cases.

The author's extension of the plane problem concept of passive earth pressure to the three dimensional problem of pressure against a pile is theoretically unjustified; formula (2), thus, can be regarded as approximate at best. Another factor adding to the difficulty of the problem is that, of necessity, the borings are made prior to the soil being disturbed by the pile driving operation. The writer contends that, in many cases, pile driving strengthens a cohesive soil considerably by the consolidation achieved by the reduction of the voids.

The writer believes that, despite its limitations, the load test to failure is the best means available to evaluate a pile's capacity or the safety factor for a given length of pile. For this reason, the author's method will be applied to some load tests made at Langley Field, Virginia, for the National Advisory Committee for Aeronautics (Figure 1). The writer calculated the h-value required to safely carry a 30-ton load by both the author's formulas (11) and (12). An h of 24 feet was required if the soil was considered cohesive and an h of 45 feet for non-cohesive soil. This results in a minimum pile length required by the author's method of 27 feet whereas a pile length of 14 feet was load tested to 84 tons in the field without failure. If an 84-ton load is conservatively assumed to constitute failure for a h of 11 feet, a lower limit can be determined for the safety factor in the author's method by solving formula (11) for P with h equal to 11 feet. The recommended allowable P resulting is approximately 15 tons. The minimum safety factor, as applied to the tested soil at Langley Field, is:

$$\frac{\text{Ultimate P}}{\text{Allowable P}} = \frac{84}{15} = 5.6$$

It is evident that the blind use of the author's method for the above project would have resulted in a more costly foundation than necessary. It is for this reason that the author cautioned that the method "be used with engineering judgment - not in lieu of it."

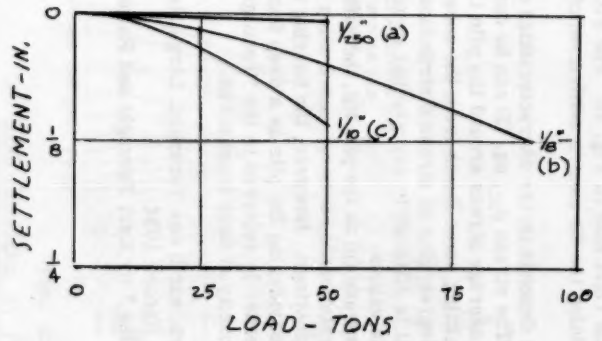
The author's work, the outline of a systematic design procedure and the correlation of the many variables in useful graph form, should be a valuable aid to the engineer interested in setting up a design procedure to fit an empirical formula which he has derived to be compatible with load tests in his area.

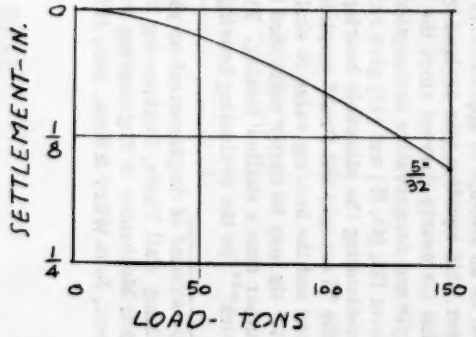
C. Y. TENG,[22] A.M. ASCE.—These equations are relatively simple and readily applicable to the ordinary pile designs. All required information is furnished by more or less standard m...mum soil exploration, and no extra tests are mandatory. Therefore, these equations are comparable to those in ordinary engineering practice.

Equation (1) is based on the assumption that the skin friction against piles in clay is equal to the cohesion of the clay. This assumption has been proved by test piles in plastic clays. However, the pile driving tends to destroy the grain structures of some sensitive clays and consequently reduces their cohesive strength in the vicinity of the piles. The amount of reduction in cohesive strength can be determined by simple soil tests. In large cities experience has usually indicated whether or not the soils are sensitive.

The author has derived very useful equations for piles driven in non-cohesive soils. He assumed that the soil surrounding the pile is in the passive state and, therefore, proposed the use of the value $\text{Tan}^2(45^\circ + \phi/2)$ for the

22. Design Engr., Sargent & Lundy, Chicago, Ill.





Individual Pile Tests

Pile "a" - h = 18 ft.
Pile "b" - h = 11 ft.
Pile "c" - h = 18 ft.

3-Pile Cluster Test
h = 15 ft.

All Piles - Raymond Std.
Cut-off El. - +5.0 ft.

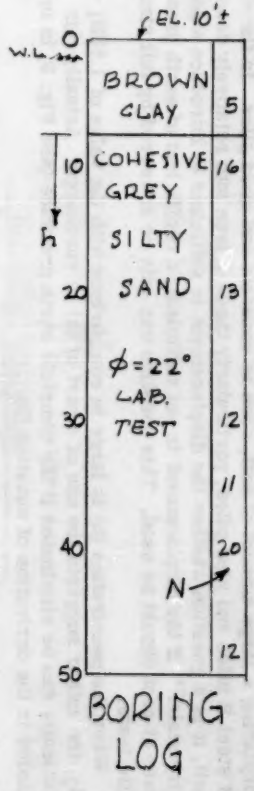


FIG. 1- LOAD TESTS

553-15

coefficient of earth pressure. This assumption seems compatible for piles displacing a large amount of soil, such as concrete and wood piles. In the case of steel H piles and pipe piles, particularly those driven into relatively dense soil, it is a question whether the displacement is sufficient to introduce a passive state.[23] If the displacement is not sufficient, a coefficient of earth pressure at rest should be used. This question can only be answered by full sized pile tests.

When the penetration (h) is large in comparison with the value of $1.58H_1 + H_2$, the author suggests the use of a part of (h) as surcharge. Actually this difficulty can be eliminated if the complete earth pressure (see Fig. 9) is included in the derivation of equation (2).

$$oh = \sqrt{(1.58H_1 + H_2)^2 + J(P - Pa)} - (1.58H_1 + H_2) \qquad (2A)$$

where

$$J = \frac{63.5}{\text{Tan}^2(45^\circ + \phi/2)\ \text{Tan}\,\phi}$$

The value of J can be approximated from the following table:

N (No. of Blows per foot)	10	20	30	40
J	40	30	20	15

The skin friction, after being transmitted from the pile to the surrounding soil, induces stresses in the soil of various intensities, see Fig. 10 left side. Theoretically the stress in the soil immediately adjoining the pile tip is infinitely large. However, as soon as the stress in the soil approaches its shear strength in situ, the soil begins to yield excessively and the stress is extended to the adjacent zone. Therefore, the actual stress distribution should be somewhat similar to the dotted line shown on the right side of Fig. 10. The excessive yielding of the soil immediately adjoining the tip causes settlement of the pile.

The actual curve of stress distribution depends on the characteristics of the soil, the magnitude of pile load, etc. The stress p_m, eq. (3) can be considered as an attempt to approximate the average stress around the pile tip. However, this equation was derived by omitting some items from the correct equation (or nearly correct since the 60 degree cone of stress distribution is a simple and conservative assumption). It is difficult to say whether or not equations (3), (4), (5) and (13) give reliable values.

In estimating the allowable bearing pressure (p) at the pile tip, the author used the "Terzaghi and Peck" empirical relationship between penetration resistance and the bearing value of shallow footings. However, the bearing value of a pile tip may be better estimated by considering the pile as a deep foundation rather than a shallow footing. The reader is referred to the "Terzaghi and Peck"[24] for the evaluating bearing capacity of deep foundations.

23. For amount of displacement or yield vs. earth see Terzaghi, Large Retaining Wall Tests, Engineering News Record 1934.
24. "Soil Mechanics in Engineering Practice," by Karl Terzaghi and Ralph B. Peck, John Wiley & Sons, Inc., 1948.

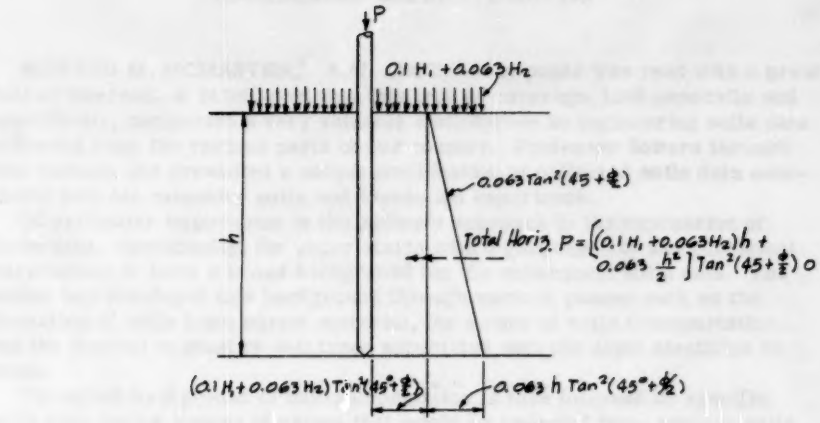


FIG. 9
TOTAL HORIZ. PRESSURE AGAINST PILE IN SAND

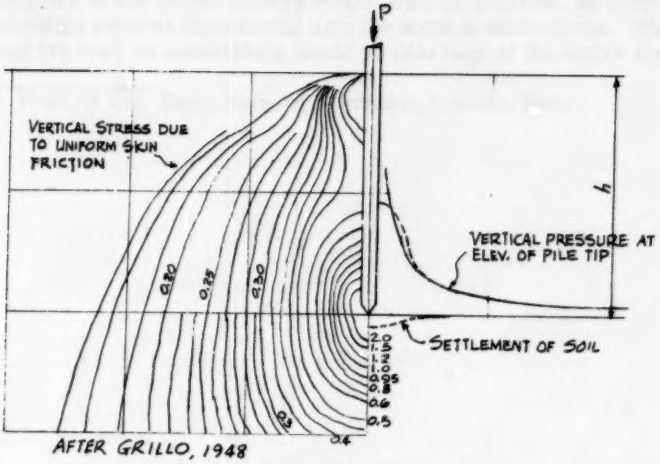


FIG. 10
SOIL PRESSURE AT PILE POINT

DISCUSSION OF SOIL PROBLEMS IN THE SOUTHERN PIEDMONT REGION PROCEEDINGS—SEPARATE NO. 416

HOWARD M. MCMASTER,[1] A.M. ASCE.—This paper was read with a great deal of interest. It is believed that this type of coverage, both generally and specifically, comprises a very valuable contribution to engineering soils data collected from the various parts of our country. Professor Sowers through this medium has presented a unique combination of collected soils data associated with his extensive soils and foundation experience.

Of particular importance is the author's approach to the expression of these data. Specifically, the paper starts with a geographical and geological introduction to form a broad background for the subsequent soils data. The author has developed this background through various phases such as the formation of soils from parent material, the means of soils transportation, and the general account of soil types associated with the depth identified by zones.

The broad background of basic information is then followed by specific soils data listing ranges of values that might be expected from various soils tests and the action of foundations under specific loading conditions.

Although the soils data is informative and useful, it is felt that the outstanding contribution of this paper is the collection of information, combined with experience for a specific region of the United States. This type of paper is of great importance to all engineers who may at some time have a job in this particular locality. Additional papers patterned after this one and covering other areas of our country would, without question, be most valuable in providing general engineering data for soils in these areas. The ultimate goal for such an undertaking would be coverage of the entire United States.

1. Prof. of Civ. Eng., Univ. of Nebraska, Lincoln, Nebr.

DISCUSSION OF FILL UTILIZATION FOR BUILDING FOUNDATIONS
PROCEEDINGS—SEPARATE NO. 417

DALE T. HARROUN,[1] A.M. ASCE.—The author has made an excellent summary of an important phase of soil engineering. The practice of placing building foundations on properly constructed earth fills has become increasingly prevalent during the past fifteen years. Since the technique of soil compaction has been an emperical development[2] used originally for earth dams and highway embankments, engineering data on additional uses must also come from the accumulation of experience.

The writer was interested in the author's general statement on the delayed time-settlement characteristic of compacted clay fills. (Last paragraph 417-6). Since soil compaction by impact is an excess pore pressure phenomenon, the soil mass does not reach complete internal equilibrium until the excess pore pressure has drained off and the moisture films on the soil fines have arranged themselves into stable patterns. In the case of a clay soil, considerable time might be required. This slow adjustment of the soil structure might explain some of the puzzling effects noted in compacted fills, such as the time-settlement effect mentioned by the author.

The writer has been observing a fifty-foot compacted earth fill which was placed two years ago. During the last year the fill has shown a settlement of 0.2 ft. This is not much percentagewise but still could be undesirable for a building foundation. High earth dam practice[3] of compacting soil under rollers sufficiently heavy to preconsolidate well above the surcharge load might be an answer. However many soils, such as some of the decomposed mica schists, will not take heavy compaction.

Another related effect has been noted by the writer in the setting or hardening of some fills with time. This amounts to a reversal of the thixotropy found in the remolding of natural clays. A controlled fill on an airport project with which the writer was connected was composed of a poorly graded silt with some clay. This fill showed a decided tendency to spring under heavy equipment at the time, yet a year later the fill was so hard and compact that an electrical contractor's men were obliged to use adzes to chip out a trench, even though there was not appreciable change in water content.

In the fourth paragraph, page 417-7, the author refers to the typical dish-shaped depression found in soft plastic soils supporting a fill and building loads. This type of settlement is not peculiar to soft plastic soils but occurs to a greater or lesser extent in any soil layer carrying a uniform load of large area, a power house mat for instance. The settlement pattern can be computed as a function of the increase of vertical pressure throughout the

1. Prof. of Soil Mechanics, Univ. of Pennsylvania, Philadelphia, Pa.
2. Design and Construction of Rolled Earth Dams. Engr. News Record by P. R. Proctor, Vol. III (1933) pp 245-248, 286-289, 348-351, 372-376.
3. The Use of Heavy Equipment for Obtaining Maximum Compaction of Soils, by O. J. Porter, Technical Bulletin 109 (1946) American Road Builders Association.

soil mass by means of Boussinesq's equation or one of the graphical solutions.

The writer has had the opportunity of inspecting a long narrow footing 6 ft. wide by 60 ft. long which rested on a deep layer of highly consolidated residual silty clay. The footing carried a substantial load of spinning machinery which was operated by a long shaft. Due to the differential movement between the center of the footing and the two extremities a continual program of shimming was required to maintain alignment.

A subsequent design relieved this situation by narrowing the bearing area beneath each end so as to build up and contact pressure locally and increase the end settlements.

This sagging settlement curve can be illustrated graphically for the case of a long narrow footing by assuming a 60^{o} pressure distribution and plotting settlement as a function of increase of vertical pressure. See the writer's Figure 1.

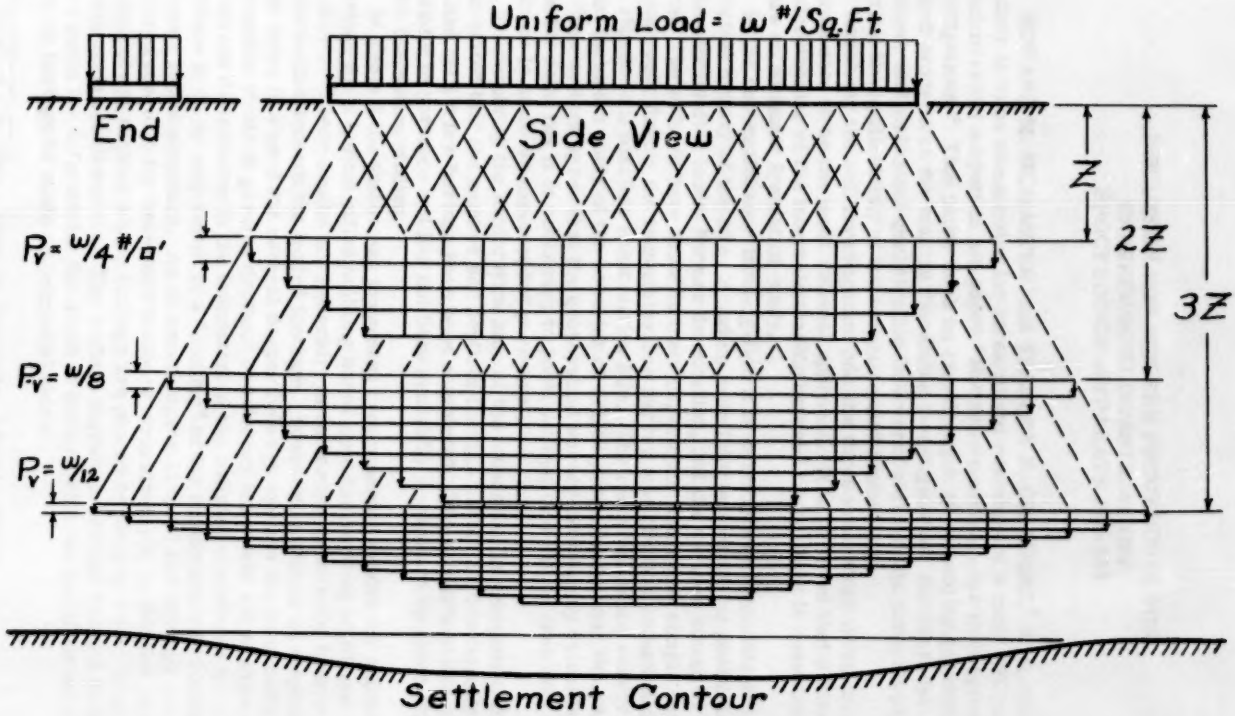
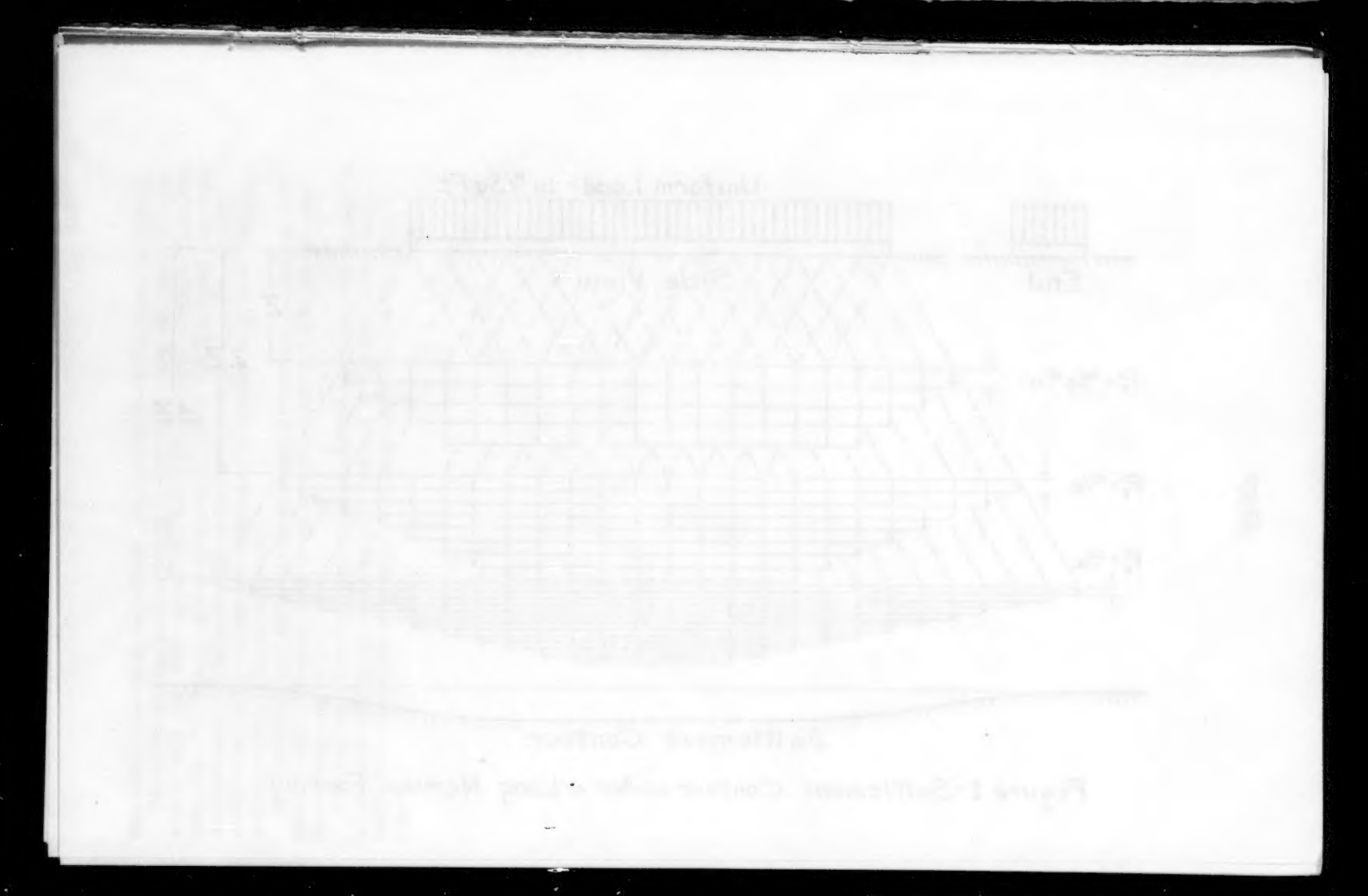


Figure 1-Settlement Contour under a Long Narrow Footing



DISCUSSION FOR MODERN PROCEDURES FOR UNDERGROUND INVESTIGATIONS PROCEEDINGS—SEPARATE NO. 435

HOWARD M. MCMASTER[1] and STANLEY F. GIZIENSKI,[2] A.M. ASCE.—The author is to be commended for an excellent coverage of a most difficult and controversial subject in his paper "Modern Procedures for Underground Investigations." This paper does an excellent job of presenting generally recognized procedures for making foundation investigations. However, the writers believe that additional qualification in several parts of the author's paper will extend the applicability of the recommended procedures.

Resolving the soil investigation into the three main steps as outlined is considered to be the most rational approach. It is believed that a thorough understanding of the interrelationship between these steps is essential to assure an adequate foundation study.

In the reconnaissance investigation it is desirable to formulate a brief geological history of the site, including such information as the possibility of preconsolidation loads, former dessication, and the various means of deposition. The writers have found in their experience that a few simple hand auger holes made during the reconnaissance survey may provide enough information to eliminate a possible construction site. The reconnaissance survey should also include a study of the behavior of existing structures near the site.

The writers agree that the geophysical survey may supply valuable supplemental data, but it is necessary to point out that this can be true only when the survey is made by experienced personnel.

In discussing the exploration step of the foundation investigation, the author has stated that the number and depth of borings cannot be intelligently determined until a few borings have been completed. The author's statement is of definite importance and has not been generally recognized by those unfamiliar with foundation studies.

In Table 4 the author has presented recommended depths for exploratory borings for a typical office building based upon the building width and number of floors. These depths are generally in conformance with the theory of stress distribution beneath the loaded footings. However, it should be emphasized that there is a need for several deeper holes to explore the possibility of compressible strata at greater depths. The deeper holes will aid in examination of stress distribution in the foundation of the building considered as a whole. Failure to do so may result in a "dishing" of the foundation with consequent damage to the structure. As an example, the 11 to 12 foot borings are not considered adequate for one-story masonry structures 100 to 400 feet in width, especially in regions where footings are placed as deep as 5 feet to meet frost penetration requirements. The writers suggest a revised Table 4 to be used as a guide for determining the depth of borings with the qualification that a few deeper borings be made as suggested above.

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1. Prof. of Civ. Eng., Univ. of Nebraska, Lincoln, Nebr.
2. Civ. Engr., Corps of Engrs., Omaha, Nebr.

TABLE 4 (Revised)

Depth for Exploratory Borings

Building Width	Number of Floors (Typical Office Building)				
	1	2	4	8	16
100 Feet	20	30	50	100	100
200 Feet	20	30	75	150	150
400 Feet	20	30	100	200	200

Examination of Table 5 indicated that some change in the columns: Applicability, Soil Disturbance, and Recognition of Changes in Soil, will provide more general coverage. Boring experience indicates that Table 5 should be revised as follows:

TABLE 5 (Revised)

Drilling Methods Suitable for Exploration

Method	Applicability	Soil Disturbance	Recognition of Changes in Soil
Hand Auger	All soils above groundwater and cohesive soils below groundwater	Considerable	Excellent
Power Auger	All soils above groundwater and cohesive soils below groundwater	Considerable	Good to Fair
Wash Boring	All soils except gravels	Very much	Poor
Rotary Drilling	All soils except gravels	Considerable	Fair
Percussion Well Drilling	All soils	May be Considerable	Fair
Coring with a Tube	All soils except course gravels	except sensitive or loose soils	Excellent if coring tube is short.

The author suggests that the penetration resistance coupled with the results of simple soil tests such as: liquid and plastic limits, water content, and grain size, may be used in the design of small structures. The writers are in general agreement with the above statement. The author further suggests that Figure 2 be utilized to determine the allowable footing pressure based upon penetration resistance. It would be desirable to know if the author had considered the possibility of differential settlements in developing Figure 2. It is interesting to note that in areas where clay foundations are encountered, shear failure of the foundation is not as critical as differential settlements, and in such cases allowable bearing pressure, based on shear strength alone, are entirely inadequate. The general applicability of Figure 2 is in doubt without further information about the factors considered in its development.

In the exploratory step of the foundation investigation, it is sometimes

desirable to take continuous undisturbed samples from one hole at a small additional cost. If the rest of the borings show uniform stratification, the representative undisturbed samples on the more critical strata may be tested. In this manner the detailed investigation would be completed, thus eliminating a second trip of the drilling rig and crew to the site. Of course, if rather erratic conditions are encountered, additional undisturbed sampling or field testing may be required.

In discussing the three principal steps in conducting a foundation investigation in the United States, the author has justifiably pointed out that the best one is the one whereby the designing organization undertakes the entire job of underground investigation. The writers' experience shows that this method has been very successful in the Corps of Engineers because of the heavy volume of work involved. The writers agree with the author that most private engineering firms do not have a sufficiently steady volume of work to justify maintaining boring equipment, and laboratories, and therefore, must look to outside help. One of the most important advantages of the "package investigation" is that the person or persons charged with the responsibility of the analysis can follow the entire sequence of sampling and testing, thus assuring accurate and comprehensive results. It should be emphasized here that the correct planning of explorations and subsequent analysis of the results of soils tests requires engineers especially trained and experienced in theoretical and applied soil mechanics.

In summary, the writers believe that the author has presented an excellent paper that combines easy reading with a clear and concise approach to a most difficult subject.

W. R. JUDD,[3] A.M. ASCE.—This paper presents some of the factors necessary to develop an underground investigation program in soils; however, any attempt to tabularize engineering geology or soil mechanics, such as is done in this paper, may have undesirable results. It usually is an author's intent that tabular data should be used only when the qualifications in the accompanying text are thoroughly understood. Experience has shown that a tabular presentation is so easily read and used that the appended qualifications are soon forgotten. Therefore, I cannot agree that the rather comprehensive field of geotechnics can or should be simplified into 9 pages including 8 tables. In general the paper would, I believe, be less subject to criticism if the author had explicitly stated that he was discussing soil investigation procedures. As it stands, the implication can be drawn that the text covers both soils and rock, an implication with which this discussion disagrees. Furthermore, some of the tabular data seems to be somewhat contrary to personal experience.

Table 2 is of particular interest and much of the data appears to be well-founded. However, the total of 0.5% for dam explorations appears to be more realistic than the 0.1% figure, particularly for large dams founded on complex overburdens or highly faulted rock with a complex geological situation.

Table 3 presumes a grid type of exploration program for the various structures mentioned. This type of program is suitable, ordinarily, for a large structure such as a spillway or possibly a multistory building. However, some difficulty may be encountered if this type of spacing is used for explorations for earth or concrete dams, due to the appurtenant structures connected with the dam. Very often, spacing at the intervals recommended in the table would result in overlapping of holes or an insufficient number of holes to fully

3. Eng. Geologist, Office of the Chf. Engr., Bureau of Reclamation, U. S. Dept. of the Interior, Denver, Colo.

explore geological conditions. Furthermore, the length of the dam would have a great influence on the spacing of the holes, regardless of soil conditions: For a dam several thousand feet long, borings 100 feet apart may be too frequent (if the subsurface geology is relatively simple) and therefore uneconomical; for a short dam, say 300 feet long, ordinarily a sound exploration program should involve more than 3 drill holes for average foundation conditions. The recommended spacing apparently does not take into account upstream or downstream explorations for the heel and toe of the dam. The boring spacing recommended for buildings may not always apply; e.g. in a multistory building, a few of the columns may carry heavy loads to the foundation and therefore holes will have to be placed at these column locations, regardless of any predetermined borehole spacing. The spacing of boreholes, therefore, should be dependent more upon the structure outline, the location of columns in the case of buildings, the location of appurtenant structures in the case of dams, and most important, upon the relationship between the structure and the geological situation.

Some of the above qualifications would apply to Table 4. If a building was only one story high but built of heavy material such as reinforced concrete with a concrete roof and highly concentrated column loads, it probably would be necessary to carry the holes deeper than 11 feet to fully explore all soils within the zone of influence from the load. It is realized that the author implies increased hole depths under such conditions in the text matter appended to the table; however, the table by itself could be misleading. A geological and structural engineering analysis should be made during the reconnaissance for the structure before the final depth of hole is determined.

Some of the information in Table 5 was difficult to understand due either to typographical errors or to a lack of sufficient explanation. It is assumed that the statements made under "Soil Disturbance" refer to the disturbance of the soil around the hole and not the disturbance to the sample itself. If this is true, exception would have to be taken to the statement concerning wash boring: it has been the writer's experience that considerable disturbance occurs to the materials adjacent to the hole when wash boring, particularly in silts and sands and gravels. If that particular column refers to the amount of disturbance to the sample, then some of the statements appear to be somewhat contrary to experiences: in hand and power auger holes, and wash boring, there is considerable soil disturbance.

With regard to the column entitled "Recognition of Changes in Soil," it is true that, generally, changes in soils can be recognized easily when using a hand auger (because of the short penetration by each lift of the auger). However, with a power auger, if of the continuous flight type, it is very difficult to identify the depth at which the soil horizon changes: due to the slow and irregular travel of the soil up the auger flight, it often is impossible to tell exactly from what depth the soil has come that appears at the surface. This is particularly true in highly cohesive soils such as clay; since clay tends to adhere to the auger, it can only be examined when the entire flight is removed from the hole. Furthermore, the soil adhering to the auger may be contaminated by soil from the walls higher up in the hole.

Many of the same statements hold true for wash boring. In studying the cuttings returned by the water, it often is impossible to tell exactly from which horizon the cuttings have been obtained due to contamination by overlying soils, unless casing has been used in the hole. Furthermore, the wash boring method tends to give deceptive information on the amount of fines in the hole. Often the fines are carried away in the return flow of water quite

rapidly, whereas the coarse materials tend to settle out. Thus, examination of the coarse cuttings may lead to the conclusion that there are no fines, whereas the converse may be true.

The statement in that same column concerning rotary drilling—"Fair (too fast)" is difficult to understand. If this refers to examination of the soil cuttings, it should be noted that one of the major methods for logging oil well holes is by examining the cuttings; and as exact determination can be made of the time required for these cuttings to reach the surface, the location of a particular stratigraphic horizon can be established to within a few inches. If a core barrel is used, then analysis of the cuttings becomes unnecessary, and stratigraphic horizons are easily located by studying the drill core in the barrel.

In percussion well drilling, the same statements made with regard to wash boring would hold true. Generally in percussion drilling, water is poured into the hole, and the percussion bit pounds up and down. Then, a bucket or bailer is lowered into the hole and the cuttings at the bottom of the hole are bailed out. Many of the coarse materials may have settled out in the hole, and thus only the fines will be secured in the sample. Certainly, as the author states, the most widely used and the most effective means of recognizing soil changes is with a drive sampler. The writer believes that much of the information presented in this table would have been more understandable if the author had explicitly stated that it referred to soil and not to rock sampling procedures.

The information in Table 8 is sound when evaluated according to the statements made in the text. Unfortunately many persons use such tables with complete disregard for the correlative laboratory testing which is absolutely necessary. The table and descriptive text make no mention of the difficulties encountered with the penetration resistance spoon in testing certain soil types.

1) Highly expansive clays may give a very high unconfined compressive strength according to the penetration resistance test, whereas the expansive characteristics are left unknown, unless laboratory tests are performed.

2) Loess, and certain modified forms of loess and silt, may have a very high penetration resistance in the dry state. However, it has been found that in loess, although the penetration resistance may indicate bearing values as high as 5,000 pounds per square foot, actually the soil may bear only 500 pounds per square foot when saturated. (This latter condition is most likely to occur around structures associated with water projects, such as hydroelectric plants, dams, and spillways. Furthermore, when residences are founded on silts with these critical hydroconsolidation properties, the owner has found that after lawn watering is started, the silts start to compress as much as seven per cent and settlement of the house foundation occurs.) A laboratory consolidation test would have indicated the compressible nature of the loess, if the consolidation test had been saturated during the loading cycle.

3) If a soil contains occasional gravel particles, the penetration spoon may give a blow count much higher than would be indicative of the actual bearing capacity of the overall soil mass; this could be true, for example, in a GC (gravelly clay) soil; the overall mass of clay may have a fairly low strength, whereas the gravel will interfere with the penetration of the spoon and a misleading high blow count will result.

Emphasis should be given to the author's comments that in any case where penetration resistance tests are used, it is absolutely necessary to have laboratory evaluations of these same materials. Once representative samples have been tested in the laboratory, then the penetration test can be used safely to

fill in the "blank" spots in the subsurface picture. Thus, it is not necessary to perform laboratory tests on every sample obtained by the penetration spoon. Furthermore, recent research[4] has shown that the length of drill rods, amount of surcharge on the soil, and degree of saturation of the soil all can effect the relationship between the penetration resistance and the unconfined compressive strength.

It generally is pointed out in the text, that the foundation explorations for any type of structure should be carried out by competent personnel and should include an adequate amount of both laboratory and field testing. However, tables can become so easy to use that some engineers may take unwary shortcuts with sad results. The many problems inherent to soil mechanics and geology make it advisable for specialists in these respective fields to conduct such programs.

CARL W. FENSKE,[5] A.M. ASCE.—The soils beneath a structure must carry the loads transmitted to them by the structure just as the structure itself must be capable of withstanding the loads applied to it. The determination of the nature and the load-carrying capacities of the foundation soils should be just as important as any phase of the structural design. The concise but complete summary of subsurface exploration procedures presented in this paper should be a valuable aid to all engineers concerned with the problem of obtaining adequate information regarding foundation soils.

The author has divided soil investigation into three steps. The first step, the reconnaissance investigation, has been covered very adequately. In passing, it should be mentioned that geophysical methods are usually suitable only when approximate information about large areas is required. The writer believes, however, that the next two steps, exploration and detailed examination, can frequently be combined. This combination is necessary in many cases when the time available for the investigation is limited or when boring equipment has to be moved a great distance. Combination of the two steps is possible by making all borings as undisturbed sample borings. It is the writer's experience that the use of a 3-inch Shelby tube sampler and a modern rotary drilling rig capable of forcing the sampler into the ground by hydraulic pressure will produce samples that are suitable for any laboratory testing that may be required. This type of boring and sampling can be made quicker and at only a slightly greater cost than the usual boring with split-spoon sampling. An inherent and unique part of this combination investigation begins with the removal, at the site, of the Shelby samples from their tubes. A trained technician classifies each sample as it is removed, and seals a selected portion for possible laboratory testing. A hand penetrometer may be used as part of the field procedure to get an indication of the variation in consistency in cohesive soil deposits when preliminary or low-cost studies are appropriate. Experience has shown that the penetrometer readings are more easily analyzed and give a better indication of the variation in consistency than do the penetration resistance values for the split-spoon. For cohesionless soil deposits, the 3-inch Shelby tube sampler can be driven to obtain results comparable to the penetration resistance of the split-spoon sampler. An extensive investigation was made for a steel mill on the Ohio River in Kentucky where many driven Shelby tube and split-spoon samples were taken in a thick deposit

4. "Second Progress Report of Research on the Penetration Resistance Method of Subsurface Exploration," U. S. Bureau of Reclamation Earth Laboratory Report No. EM-356, Oct. 5, 1953.
5. Associate Engr., Greer & McClelland, Cons. Foundation Engrs., Houston, Tex.

of clean sand. Experience gained on this investigation has shown that penetration resistance of a standard 2-inch O.D. split-spoon sampler under blows of a 140-pound hammer dropped 30 inches and the penetration resistance of a 30-inch O.D. 2.83-inch I.D. Shelby tube sampler driven by a 200-pound hammer dropped 24 inches are practically identical. When this procedure is used, suitable samples are available from every boring for a study of any degree of detail necessary and it is not necessary to go back and make additional undisturbed sample borings as is necessary when split-spoon sampling is used.

The data from a split-spoon sample boring must be analyzed with care as the penetration resistance of the sampler depends on many factors other than the consistency or denseness of the soil. Because of this, correlations of the penetration resistance with pertinent soil properties show a large variation. On occasion, these other factors have such an effect that the penetration resistance values lead to misleading results. The following statement, quoted from an article by A. Mystkowski concerning the foundation conditions at the site of a paper mill in Georgia[4], illustrates this point:

> "Although various field and laboratory investigations were called for, relative density tests were considered to be the only foolproof ones. It was practically impossible to find any correlation between relative density and the number of blows on standard penetration tests. Even in clean, uniform fine sand, without any fines passing through 200-mesh sieve, the recorded number of blows varied between 10 and 20, while relative density tests showed all densities above 75%. On the other hand, in some cases fine sand of relative density below 50% required over 30 blows. In layers with even a small clay content—2% to 5%—this inconsistency of the standard penetration tests was accentuated."

This variation, for clays, is indicated in the correlation given by the author in Table 8. Using the average values in this table, a penetration resistance of 5 blows per foot, for example, in a clay deposit would indicate an unconfined compressive strength of 750 to 2500 psf, depending on the type of clay in the deposit. If the deposit was determined to be a homogeneous, highly plastic clay, a penetration resistance of 5 blows per foot would correspond to an unconfined compressive strength of from 1500 to 3500 psf. This variation in values has led to the following statement by Karl Terzaghi, Hon. M., ASCE:[5]

> ". . . .the consistency of clays has been correlated with the number of blows per foot. However, in connection with clays, experience has shown that the scattering of the results can be very important. In other words, at a given number of blows per foot of penetration the consistency of the clay (unconfined compressive strength) can vary between rather wide limits. Hence when dealing with clays, it appears preferable to estimate the average consistency on the basis of the results of unconfined compression tests."

The determination of the pertinent soil properties from laboratory soil tests would mean an additional cost which experience from a large number of investigations has shown to be roughly one-third the cost of the borings. This additional cost of determining the soil properties from laboratory test is usually more than compensated for by the savings in the foundation design resulting from a more definite knowledge of the pertinent soil properties. When a number of unconfined compression tests are made from samples from all borings and the minimum strength value thus determined is used, experience has shown that a factor of safety of two may be used to compute the allowable

bearing pressure. If a correlation such as that in Table 8 is used, a factor of safety of at least three is recommended and the prudent designer would be inclined to use the minimum strength value given by the table. The spending of a little more money for undisturbed sampling and laboratory testing is justified by the more economical foundation design. If $1,000 is saved by the expenditure of an additional $500, then the expenditure is justified even though the percentage cost of the investigation is increased.

Summarizing these comments, the use of the procedures outlined in the preceding paragraphs will produce subsurface information and suitable samples for an investigation of any size or to any degree of detail. Samples suitable for any type of laboratory testing are available from every boring. The pertinent soil properties can be determined from the results of soil tests even for the small job. The actual cost of an investigation of this type may be slightly greater than one in which only split-spoon sampling was used but the additional cost is usually more than compensated for by a more economical foundation design. If the soil conditions are found to be such that a detailed examination is necessary, the investigation will have been made at a somewhat lesser cost than if split-spoon sampling and subsequent undisturbed sampling had been used. However, if this procedure is followed, the field investigation should be under the direct supervision of a trained soils engineer. His control over the borings and sampling, laboratory testing, and analysis of the data should be such that any phase of the investigation can be modified at any time to meet any conditions disclosed by the investigation. This requirement is satisfied when a competent engineering firm specializing in this procedure is retained to make what the author describes as a "package" investigation.

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